C6.2 Piles

C6.2.1 General

C6.2.1.1 Policy overview

Methods Memo No. 79: Integral Abutment Piles
24 July 2003

See C6.5.1.1.1.

C6.2.1.2 Design information

C6.2.1.3 Definitions

C6.2.1.4 Abbreviations and notation

C6.2.1.5 References

C6.2.2 Loads

C6.2.2.1 Dynamic load allowance

C6.2.2.2 Downdrag

Methods Memo No. 140: New Plan Note E175/M175, "Waiting Period for Driving Piles"
2 November 2005

See C11.3.2.

C6.2.3 Load application

C6.2.3.1 Load modifier

C6.2.3.2 Limit states

C6.2.4 Analysis and design

C6.2.4.1 General

Methods Memo No. 23: Length Limits and Prebore Depths for Integral Abutment Bridges
30 October 2002

See C6.5.1.1.1.

Methods Memo No. 14: Prebore Lengths for Integral and Stub Abutments
13 September 2001 (Note that standard prebore length now is 10 feet (3.050 m). See Methods Memo No. 23 in C6.5.1.1.1.)

When determining pile lengths for integral and stub abutments, downdrag forces may need to be included in the design. To help reduce the effects of the downdrag forces, the designer may consider increasing the prebore depths (standard length 8 ft. (2400 mm)) for the integral abutment piling or providing prebore for stub abutments piling.

If the prebore length is increased, the maximum prebore lengths should be as follows.

1. For integral abutments, a maximum prebore length of 15 ft (4500 mm).
2. For stub abutments, a maximum prebore length of 20 ft (6000 mm).

These lengths are based on the office's experience with longer prebores and stability checks of the piling using the following assumptions:

1. A maximum deflection of 1 ½ inch (40 mm) or 3 inches (75 mm) total movement for the integral abutments. Note: This deflection includes setting factors of 1.5 for prestressed beams and 1.33 for steel girders.
2. No lateral deflection of the stub abutment.
3. Using HP10 x 42 (250 x 62) steel piles with a maximum bearing of 55 tons (489 kN)
4. No lateral support from the bentonite slurry that is currently used as back fill in the prebore.
5. A minimum pile length of 2.5 times the prebore length. If this length cannot be reached then a special analysis needs to be done.

In situations, where the prebore needs to be increased or included in the abutment design, please check with your section leader for approval.

Methods Memo No. 9: Battered Pile Capacity and Lateral Load Capacity for Pier Design
9 April 2001

See C6.6.4.1.3.1.

C6.2.4.2 Downward load

Methods Memo No. 55: Use of Higher Pile Capacities
5 November 2001

See C6.2.6.1.

2007 LRFD pile design practice

The foundation pile design practice that has been followed by the Office of Bridges and Structures (OBS) is not covered directly by the AASHTO Standard or LRFD Specifications. Although the AASHTO LRFD Specifications ideally will be based on reliability theory, in many cases insufficient information is available for calibration. As of early 2007, "only the strength I limit state has been formally calibrated" (Barker and Puckett 2007). Many of the LRFD calibrations are by fitting to allowable stress design. That step has been taken because if one deviates from successful past practice there should be a strong reason, such as a large and reliable data base coupled with reliability theory (Allen 2005).

As a temporary measure, in view of the October 2007 deadline for conversion of bridge design to LRFD, OBS has calibrated pile design practice to LRFD by fitting with the 1994 soils information charts. Experience with the charts has been favorable and, to avoid disruption in the field, the office also intends to continue placing the same required bearing information on bridge plans. In 2007, research was initiated through the Iowa Highway Research Board (TR-573, TR-583, and TR-584) to examine pile design calibration with respect to reliability theory, as well as to examine present design and construction procedures. The research will take two years or more. After completion of the research it is likely that there will be both design and construction policy changes.

The current policy in the LRFD manual [BDM 6.2.4.1] is based on the following:

- An average load factor, γ_{bar} , of 1.45, for a dead to live load ratio of 3:2. This factor is reasonable for the Strength I limit state and was used in a recent National Highway Institute LRFD substructure course (Withiam et al. 1998), but the factor is not universally accepted because a more recent study used a dead to live load ratio of 3:1 (Allen 2005), which results in an average load factor of 1.38. The 1.45 factor fits well with typical Iowa integral abutment pile loads under the Strength I limit state but varies from the typical 1.70 determined from pier analysis studies that include other strength limit states. Thus it is likely that the number and/or size of piles will increase for pier design under this initial calibration of pile design practice.
- A continuation of present practice limiting service H-pile axial stress to 6 ksi for piles in friction bearing and to 9 ksi for maximum end bearing on bedrock, except for special situations for which the axial stress may be increased to 12 ksi. Generally maximum end bearing of 12 ksi is permissible only for hard rock with N > 200, and other stresses are applicable for specific conditions defined in the soil information charts or defined in the manual for H-piles [BDM 6.2.6.1].
- A minimum factor of safety of 2.0 in the 1994 soils information charts. Because the information in the charts was developed by different methods, the estimated factor of safety was greater than 2.0 in some cases. The 2007 LRFD friction and end bearing charts [BDM 6.2.7] have all values increased by a factor of 2.0 to remove the safety factor so that the charts give nominal resistance values. The new charts also have been converted to kip units, the standard units for the AASHTO LRFD specifications. Thus, with the exception of end bearing stress values, the values in the new charts are four times the values in the 1994 charts used for service load design.
- An H-pile structural phi factor, φ_c, of 0.6, which considers axial compression in the lower part of the pile. The Soils Design Section may lower this phi factor to 0.5 for unusually severe driving conditions. (In cases where the designer checks a combination of forces and moments at the top of the pile, different resistance factors need to be applied.)
- An H-pile geotechnical phi factor, φ, of 0.725 obtained by fitting to the present soils charts with a formula commonly cited (Barker and Puckett 2007, Allen 2005). Tabulated values in the charts were developed by various methods, and therefore the phi factor could vary under the latest AASHTO LRFD Specifications from 0.4 to 0.9. The 0.725 phi factor falls within the overall range and, at this time, it would be difficult to obtain a more accurate value.

References

Allen, T.M. (2005). *Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design, FHWA-NHI-05052.* National Highway Institute, Federal Highway Administration. Washington, DC.

Barker, R.M. and J.A. Puckett. (2007). *Design of Highway Bridges, an LRFD Approach, Second Edition.* John Wiley & Sons, Inc. New York, NY.

Dirks, K.L. and P. Kam. (1994). *Foundation Soils Information Chart, Pile Foundation*. Soils Survey Section, Highway Division, Iowa DOT. Ames, IA.

Withiam, J.L., E.P. Voytko, R.M. Barker, J.M. Duncan, B.C. Kelly, S.C. Musser, and V. Elias. (1998). *Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures, FHWA HI-98-032.* Federal Highway Administration. Washington, DC.

Piles driven to rock

End bearing of 9 ksi is the same as 648 tsf. Generally this strength would be midrange for uniaxial compressive strength of limestone, mudstone, and sandstone and bottom range for granite [NAVFAC 1986].

In a recent paper [Serrano and Olalla 2002] the researchers surveyed the ultimate strengths of rock at pile tips proposed by several authors, developed a model, and compared it with tests. Ultimate strengths proposed by others ranged from 2.7 to 11 times the unconfined compression strength of the rock. (Generally these proposals parallel the principle that locally, when confined, concrete can withstand higher stresses than indicated by cylinder tests.) The researchers considered various factors in their model and comparisons, including unconfined compressive strength, overburden pressure, length of pile embedment in rock, and fracturing and weathering of the rock. Generally the researchers' model worked well for soft rocks (to 4.35 ksi) but overestimated ultimate bearing capacities for hard rocks.

The second edition of the AASHTO LRFD Specifications, 2000 Interim includes a 1985 method from the Canadian Geotechnical Society for determining the nominal end bearing resistance [AASHTO-LRFD-2nd 10.7.3.5]. However, the commentary states: "When this method is applicable, the rocks are usually so sound that the structural capacity will govern the design." In the third edition, 2006 Interim, the AASHTO LRFD Specifications state: "The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state" [AASHTO-LRFD-3rd 10.7.3.2.3]. Also: "Soft rock that can be penetrated by pile driving shall be treated in the same manner as soil for the purpose of design for axial resistance" [AASHTO-LRFD-3rd 10.7.3.2.2]. The commentary relies on local experience for definition of hard rock.

"Except for soft weathered rock, the structural capacity of the pile will generally be lower than the capacity of the rock to support loads for toe bearing piles on rock of fair to excellent quality as described in Table 9-7 [Hannigan et al. 2005]." (Table 9-7 indicates "fair" is a Rock Quality Designation (RQD) of 50% or more.) "The structural capacity, which is based on the allowable stress for the pile material, will therefore govern the pile capacity in many cases….Piles supported on soft weathered rock, such as shale or other types of very poor or poor quality, should be designed on the results of pile load tests" [Hannigan et al. 2005].

"AASHTO limits the maximum allowable design stress to $0.25f_y$ (which refers to the AASHTO Standard Specifications). In conditions where pile damage is unlikely, AASHTO allows the design stress to be increased to a maximum of $0.33f_y$ provided static and/or dynamic load tests confirming satisfactory results are performed" [Hannigan et al. 2005]. For Grade 36 piles these stresses are 9 ksi and 12 ksi, and for Grade 50 piles these stresses are 12.5 ksi and 16.5 ksi.

References

Hannigan, P.J., G.G. Goble, G.E. Likins, and F. Rausche. 2005. *Design and Construction of Driven Pile Foundations—Volume I.* National Highway Institute (NHI), Washington, DC.

Naval Facilities Engineering Command (NAVFAC). 1986. *Soil Mechanics Design Manual 7.01.* Alexandria, VA.

Serrano, A. and C. Olalla. 2002. "Ultimate bearing capacity at the tip of a pile in rock—part 2: application." *International Journal of Rock Mechanics and Mining Sciences,* Volume 39, Issue 7, 847-866.

C6.2.4.3 Downdrag

The major issue when designing for downdrag is computation of the downward load. In the third edition of the AASHTO LRFD Specifications two cases are identified: (1) piles driven to end bearing and (2) friction piles that can experience settlement at the pile tip [AASHTO-LRFD 10.7.1.6.2]. In the first case, the specifications require that downdrag be considered at the strength and extreme event limit states. Because the extreme event limit state involves seismic design generally not required in Iowa, piles need only be designed at the strength limit state.

In the second case, the AASHTO LRFD Specifications add the service limit state because of the potential for settlement. In the past the Office of Bridges and Structures has depended on the Soils Design Section for settlement analysis and intends to continue doing so.

In both the AASHTO LRFD Specifications and office practice the computation of the downdrag load has been made less conservative in recent years [OBS MM Nos. 20 and 87]. In the second edition of the AASHTO LRFD Specifications, the downdrag load factor at the strength limit state was to be taken at 1.8. In the third edition, 2006 Interim, the downdrag load factor has been lowered to 1.05 and 1.40 depending on the method used to determine the force [AASHTO-LRFD Table 3.4.1-2]. In the calibration of these load factors, analysis for specific conditions gave factors in the 1.00 to 1.65 range [Allen 2005].

In the office, Methods Memo No. 20 (2001) indicated that the downdrag load should be computed by removing the safety factor of two from the pile chart. With that procedure, one foot of pile downdrag would be supported by two feet of friction bearing (in the same soil) or an equivalent amount of bearing. Evidently the procedure was too conservative and resulted in difficulty in designing for downdrag. A subsequent memo, Methods Memo No. 87 (2004), made the design procedure less conservative by eliminating the need to remove the safety factor from the load. Thus one foot of pile downdrag could be supported by one foot of friction bearing (in the same soil) or equivalent bearing. The rationale for the change was that uncertainty in the load would be balanced by uncertainty in the capacity, and the safety factor for downdrag could be reduced to 1.0.

To follow the latest office practice in LRFD would require that the strength limit state load factor for downdrag in LRFD be reduced below 1.0 to about 0.75. The exact calibration depends on the percentage of the total load attributable to downdrag and other factors. To provide a minimum margin of safety the load factor will be taken as 1.0, which will result in numbers of piles closer to those resulting from Methods Memo No. 87 than from No. 20. The fact that the load factor is 1.0 does not mean that there is no margin of safety; the margin of safety is provided by the phi factor on pile resistance.

For a pile subject to downdrag, driving conditions will be more severe than usual. The pile will need to be driven initially through soil layers above the layer causing downdrag, as well as the layer itself, and then driven for bearing. In cases where the driving resistance at the service limit state exceeds the load that would cause an axial stress of 9 ksi, a pile drivability analysis should be completed by the Office of Construction before the bridge substructure design is finalized.

Reference

Allen, T.M. (2005). *Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design, FHWA-NHI-05-052.* Federal Highway Administration, Washington, DC.

C6.2.4.4 Uplift

For pier piles, uplift often occurs under lateral loading at the top of the pier. Except for Service Load Group 1, the present design manual based on the AASHTO Standard Specifications allows uplift on pier H-piles provided that the pile is adequately attached to the footing and that the pile has adequate friction bearing capacity for the tension load. The designer may consider a bond stress between steel pile and concrete footing of 15 psi [BDM 6.6.4.1.3.1].

Evidently it has been permissible in the Office of Bridges and Structures (OBS) to use the full compression friction bearing value for the pile subjected to uplift. At least one source, however, indicates that the allowable compression bearing value is commonly reduced by a factor of 0.75 for tension bearing [McVay et al. 1998]. The reduction for uplift is followed in the AASHTO LRFD Specifications with a reduction in resistance factor.

As in service load design for uplift, under load and resistance factor design (LRFD) both the pile embedment in concrete and the pile embedment in soil need to be checked. Very limited data is available on the bond strength between an H-pile and a concrete footing [GAI 1982]. There is, however, related data on bond strength for encased steel columns [Griffis 1992], filled steel columns [AISC 2005], and embedded beam flanges [Watson et al. 1974]. The lowest of these bond strength values is 60 psi with an associated resistance factor of $\varphi = 0.45$ [AISC 2005]. With these bond and φ values and an average AASHTO LRFD load factor of 1.45, an equivalent allowable stress would be 18.6 psi, very close to the 15 psi in the present design manual based on the AASHTO Standard Specifications.

The AASHTO LRFD Specifications give phi values for uplift but do not deal directly with resistances determined from design aids such as the Iowa DOT's soil information charts. The resistance factors for uplift are less than those for gravity loads, evidently by a factor of 0.8 with rounding to the nearest 0.05 [Wilson et al. 2005]. The maximum resistance factor for a single pile in uplift in the AASHTO LRFD Specifications is 0.60.

If the geotechnical resistance factor to be used with the 2007 pile information charts, 0.725, is multiplied by 0.8, the result is 0.58, which would round to 0.60. This phi is at the maximum for uplift in the AASHTO LRFD Specifications [AASHTO-LRFD Table 10.5.5.2.3-1].

References and Brief Summaries of Selected References

American Institute of Steel Construction (AISC). (2005). *Steel Construction Manual, 13th Edition.* AISC, Chicago, IL.

> Commentary to the specification suggests a reasonable lower bound for bond strength of 60 psi for concrete-filled hollow structural section (HSS) steel columns. For LRFD the recommended resistance factor is 0.45. (Because AASHTO LRFD generally uses higher load and resistance factors than AISC LRFD, the AISC $\varphi = 0.45$ will be conservative if used with AASHTO factored loads.) The commentary also suggests that bond stress for an encased column be ignored, but acknowledges the reference below by Griffis, in which bond stress is considered.

GAI Consultants, Inc. (1982). *The Steel Pile, Pile Cap Connection.* American Iron and Steel Institute (AISI). Washington, DC.

> The consulting firm, GAI Consultants, Inc. conducted 16 steel pile pull-out tests, most of which were designed to test tension anchors. One bare HP 10x42 test with the pile embedded 9 inches in concrete with a strength of at least 4300 psi indicated a bond failure at an average bond stress of 74 psi. A similar 10¾-inch diameter pipe pile test with 6-inch embedment indicated a bond failure at 143 psi. The authors recommended that bond stress not be considered in design.

Griffis, L.G. (1992). *Steel Design Guide No. 6, Load and Resistance Factor Design of W-Shapes Encased in Concrete.* American Institute of Steel Construction, Chicago, IL.

> For shear connection, the author first considers bond based on a paper written by Roeder. The author considers only the flange area of an embedded wide flange shape and applies a safety factor of 5 to obtain the following:
>
> average ultimate bond stress = $u = 0.9(0.09f'_c - 95)$
>
> (The result from this formula for typical Iowa DOT 3500 psi concrete is 198 psi.)

McVay, M.C., C.L. Kuo, and W.A. Singletary. (1998). *Calibrating Resistance Factors in the Load and Resistance Factor Design for Florida Foundations, Final Report.* Department of Civil Engineering, University of Florida, Gainesville, FL.

Watson, J., R. O'Neil, R. Barnoff, and E. Mead. (1974). "Composite Action without Shear Connectors." *Engineering Journal*, 2nd Quarter, 1974. pp 29-33.

> The authors conducted working-load flexural tests of two castellated beams with top flanges embedded in 4000 psi concrete slabs. At working loads, maximum bond stress was 82.8 and 88.0 psi. The authors also tested one beam for 750,000 cycles at a maximum bond stress of 25.4 psi and found no adhesion failure. For comparison, the 1963 ACI Code allowed a bond stress of 160 psi for plain bars.

Wilson, K.E., R.E. Kimmerling, G.C. Goble, P.J. Sabatini, S.D. Zang, J.Y. Zhou, W.A. Amrhein, J.W. Bouscher, and L.J. Danaovich. (2005). *LRFD for Highway Bridge Substructures and Earth Retaining Structures, Reference Manual.* Federal Highway Administration (FHWA), Arlington, VA.

C6.2.4.5 Lateral load

For service load design of abutment and pier piles the Office of Bridges and Structures has used simple guidelines for lateral load capacities. Without special analysis, for steel H-piles the allowable lateral load is 6 kips [BDM 6.2.6.1]. For timber piles the allowable lateral load is 4 kips [BDM 6.2.6.3]. In each case, for a battered pile the designer may add the allowable horizontal component.

The nominal values used by the office generally are conservative when compared with nominal values used by others and values associated with specific soil conditions, as the tables in Table C6.2.4.5 indicate and as examples for an H-pile in soft clay suggest [BDM C6.2.6.1].

In most cases, lateral loads for piles are governed by horizontal deflections at the service limit state. The ultimate load for a pile typically is in the range of 3 to 14 times the load that causes a 0.25-inch deflection [AISC 1973].

The allowable horizontal deflection often has been limited to 0.25 inch at the ground line for buildings, with more liberal limits for other structures [Teng 1962]. The second edition of the AASHTO LRFD Specifications had a limit of 1.5 inches for bridge piles [AASHTO LRFD-2 10.7.2.2], but that limit has been removed from the third edition. Unless set conservatively, the allowable deflection should be set based on the type and configuration of the structure. For example, for the typical stub abutment, the pile head deflection represents the horizontal movement of the abutment, but at the top of a pier the deflection may be amplified by differential settlement and rotation, and that amplification should be considered.

The actual horizontal deflection at the head of a pile is governed by various factors: soil density or stiffness near the ground surface, water table in granular soils, fixity of pile head, pile size and stiffness, depth of pile penetration, group effects, and type of loading. Methods for estimating horizontal deflection generally take into account the soil properties and location of water table. The pile head condition is governed by details and pile material. Standard office details provide a considerable amount of pile head fixity, and in most cases a steel or concrete pile head may be assumed to be fixed. A timber pile head, however, should be considered pinned.

Pile size is a factor, but the orientation of an H-pile is more significant because of the difference in stiffness and ultimate moment resistance from major to minor axis. Depth of pile penetration needs to be sufficient to meet assumptions for the method. Group effects are significant if piles will move in the shadow of other piles, and group effects can double or triple the horizontal deflection of a single pile. Generally long-term loading will have more effect than short-term loading.

For load and resistance factor design (LRFD) the design checks should include lateral deflection at the service limit state and moment at the strength limit state. The design could be accomplished by three methods: conservative assumed nominal capacity at the service and strength limit states, hand computation methods, or software such as LPILE. Because the simple guidelines used by the office in the past have worked well for typical bridges their use can be continued and supplemented with strength limit state guidelines.

For steel piles the minimum nominal lateral resistance of a pile at the strength limit state shown in Table C6.2.4.5 is 18 kips, which is three times the service limit state capacity used by the office in the past. The simple guideline of 6 kips would be approximately equivalent to a nominal resistance of $\gamma*6/\varphi = (1.70)(6)/(1.0) = 10.2$ kips assuming a relatively high average load factor. Thus it would seem reasonable to set the nominal resistance at the strength limit state at 10 kips or more. For steel piles the ultimate load generally is at least three times the load that causes a 0.25-inch deflection, and that observation can be used to set the strength limit at 18 kips. The LPILE example, Example (8), in the H-pile commentary [BDM C6.2.6.1] supports the observation.

No comparable information is available for timber piles. Conservatively the strength limit could be set at $\gamma*4/\varphi = (1.70)(4)/(1.0) = 6.8$, rounded to 7 kips.

Two hand computation methods are readily available in the office: (1) a method based on work by Broms [Wilson et al. 2005] and (2) a method based on work by Evans and Duncan [Brockenbrough 1997]. The Broms method will give working lateral loads based on deflection, pile structural and geotechnical capacity, pile length, and group effects. In some cases, however, the user may need to extrapolate beyond the limits of the charts provided for the method. The method covers steel and concrete piles. Results for steel piles seem not to compare well with results from LPILE software, as shown in Examples (7) and (8) given in the H-pile commentary [BDM C6.2.6.1].

The Evans and Duncan method seems easier to use and will give individual pile and group deflections and individual pile and group maximum moments. The charts provided for the method are limited to prestressed concrete piles and steel H-piles loaded with respect to the strong axis. Results seem to compare well with LPILE.

LPILE software available in the office will give the deflection, moment, and shear with depth for a single pile. Input is relatively simple, and the program will handle a variety of pile types, soil types, and soil layers. Step by step instructions for use of the program for individual piles and general instructions for use of the results for pile groups are available to engineers in the office who have attended the recent National Highway Institute (NHI) courses [Hannigan 2005 and Wilson et al. 2005].

The LPILE software supplier, Ensoft, Inc., also has a program, GROUP, that will analyze pile groups. At this time GROUP is not available to the office.

Generally it appears that the most efficient design process is to use simple guidelines updated for LRFD for typical bridges and recommend that the designer use LPILE software for conditions requiring additional capacity or more rigorous analysis. The hand computation methods can provide checks on the software.

References

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Table C6.2.4.5. Summary of Permissible Lateral Loads for Piles

Timber pile lateral loads

Allowable Load, k	Conditions	Source	Date
1.5	¼-inch deflection, medium sand or medium clay, 12-inch diameter, free head	Vanikar, FHWA-DP-66-1 (McNulty)	1956
4	¼-inch deflection, medium clay, 12-inch diameter, fixed head	Vanikar, FHWA-DP-66-1 (McNulty)	1956
4	Poor soil	Wisconsin DOT	1998
5	¼-inch deflection medium sand, 12-inch diameter, fixed head	Vanikar, FHWA-DP-66-1 (McNulty)	1956
5	Average or good soil	Wisconsin DOT	1998
9	¼-inch deflection	Vanikar, FHWA-DP-66-1 (Feagin)	1953?
10		New York DOT	1977
14	½-inch deflection	Vanikar, FHWA-DP-66-1 (Feagin)	1953?

Concrete pile lateral loads

Allowable Load, k	Conditions	Source	Date
5	¼-inch deflection, medium clay, 16-inch diameter, free or fixed head	Vanikar, FHWA-DP-66-1 (McNulty)	1956
7	¼-inch deflection, medium sand, 16-inch diameter, free or fixed head	Vanikar, FHWA-DP-66-1 (McNulty)	1956
8	12-inch precast or cast-in-place, poor soil	Wisconsin DOT	1998
8	¼-inch deflection, sand, $\varphi = 30$, 10-inch prestressed	Barker, NCHRP Report 343	1991
11	12-inch precast or cast-in-place, average or good soil	Wisconsin DOT	1998
12	¼ inch deflection	Vanikar, FHWA-DP-66-1 (Feagin)	1953?
13	¼-inch deflection, sand, $\varphi = 40$, 10-inch prestressed	Barker, NCHRP Report 343	1991
13	¼-inch deflection, clay, undrained shear strength = 1 ksf, 10-inch prestressed	Barker, NCHRP Report 343	1991
15		New York DOT	1977
17	½ inch deflection	Vanikar, FHWA-DP-66-1 (Feagin)	1953?
34	¼-inch deflection, clay, undrained shear strength = 4 ksf, 10-inch prestressed	Barker, NCHRP Report 343	1991

General pile lateral loads

Allowable Load, k	Conditions	Source	Date
1	Soft clay	Teng	1962
2	3/8 inch deflection	New York City Building Code	1968
2	Modest deflection	White and Salmon, Building Structural Design Handbook	1987
10-20	Validated by tests	White and Salmon, Building Structural Design Handbook	1987

Steel pile lateral loads

Allowable Load, kips	Ultimate Load, kips	Conditions	Source	Date
	18	HP or steel pipe filled with concrete	Minnesota DOT LRFD manual	2004
2	20	HP 10x42, ¼-inch deflection, loose coarse-grained material below water table, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
2	25	HP 10x42, ¼-inch deflection, medium stiff fine-grained material, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
3	25	HP 10x42, ¼-inch deflection, loose coarse-grained material above water table, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
4	50	HP 14x89, ¼-inch deflection, loose coarse-grained material below water table, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
4	55	HP 14x89, ¼-inch deflection, medium stiff fine-grained material, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
5	58	HP 14x89, ¼-inch deflection, loose coarse-grained material above water table, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
7		HP, poor soil	Wisconsin DOT	1998
8	23	HP 10x42, ¼-inch deflection, dense coarse-grained material below water table, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
8	48	HP 10x42, ¼-inch deflection, very stiff fine-grained material, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
8		HP 10x42, ¼-inch deflection, sand, $\varphi = 30$ degrees, water table at ground surface	Barker, NCHRP Report 343 and NSBA V.I, Ch.10	1991 and 1997
10	30	HP 10x42, ¼-inch deflection, dense coarse-grained material above water table, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
10		HP, average soil	Wisconsin DOT	1998
12	95	HP 14x89, ¼-inch deflection, very stiff fine-grained material, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
13	50	HP 14x89, ¼-inch deflection, dense coarse-grained material below water table, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
13		HP 10x42, ¼-inch deflection, clay, undrained shear strength = 1 ksf	Barker, NCHRP Report 343 and NSBA V.I, Ch.10	1991 and 1997
15		HP, good soil	Wisconsin DOT	1998
16		HP 10x42, ¼-inch deflection, sand, $\varphi = 40$ degrees, water table at ground surface	Barker, NCHRP Report 343 and NSBA V.I, Ch.10	1991 and 1997
19	70	HP 14x89, ¼-inch deflection, dense coarse-grained material above water table, free head, normal to flange	AISC, Highway Structures Design Handbook	1973
20			New York DOT	1977
40		HP 10x42, ¼-inch deflection, clay, undrained shear strength = 4 ksf	Barker, NCHRP Report 343 and NSBA V.I, Ch.10	1991 and 1997

C6.2.5 Detailing

Methods Memo No. 117: Pile Cutoff for Battered Piles
20 July 2005

The Office of Construction recently brought up an issue concerning the details used to show the tops of battered piles in pier footings. On a recent bridge project, the cutoff of the tops of the battered piles were shown as normal to the centerline of the pile. Construction's preference is to show the pile top as a level (horizontal) surface. This allows easier placement of footing reinforcement when it is placed directly above the battered piling in pier footings. The top of the battered pile may interfere with placement of reinforcing unless it is trimmed horizontal.

Therefore, in bridge plans with pier footings where the footing reinforcement is located directly above the piles, the following note should be added to the pier general notes and the battered piles detailed to show a horizontal surface.

E724/M724 "All battered pile shall be trimmed to a horizontal line to aid in the placement of reinforcing".

Prestressed piles, which are normally driven to full penetration should not be trimmed, because of the difficulty and cost of removing the corner of the concrete to provide a horizontal surface.

C6.2.6 Guidelines by pile type

C6.2.6.1 Steel H

Because of relatively high driving stresses the Office of Construction initiated a change in the steel H-pile specification from ASTM A 36/A 36M to ASTM A 572/A 572M. The change was effective in October 2005.

Methods Memo No. 55: Use of Higher Pile Capacities
5 November 2001

Existing Iowa DOT guidelines for the design of piles should be followed with the following exceptions: For steel piles that are designed for point bearing and seated in bedrock with N-values between 100-200, you may include contribution from side friction, if the soil layers penetrated consist of suitable material. The total design stress on the pile may exceed 6 ksi (42 MPa) but shall not exceed 9 ksi (62 MPa).

Methods Memo No. 160: Design Manual Article 6.2.6.1, Revised Allowable Pile Stress
6 April 2007

During the design of the I-80 bridge across the Missouri River, the issue of allowable design stresses for piling when driving to solid rock (N > 200) was raised. The piling on this project had been designed for 9 ksi (62 MPa) allowable based on the Bridge Design Manual. However, for this bridge, there was a large quantity of piling, and it was apparent that the H-pile design using the present stress limit of 9 ksi (62 MPa) was not cost effective for the site conditions. The stress limit was raised to 12 ksi (83 MPa), and the redesign reduced the piling cost significantly.

In order to avoid redesign in similar situations in the future, the Bridge Design Manual Article 6.2.6.1 has been revised to include the option for a higher allowable stress when piles are driven to solid rock (N > 200). This higher stress 12 ksi (83 MPa) will only be available when recommended by the Soil Design Section or geotechnical consultant, and approved by the Assistant Bridge Engineer.

Before using the higher value, the designer shall contact the Office of Construction and provide pile information, bottom of footing information, and soils data for a preliminary wave equation drivability analysis. If there are drivability concerns, the Office of Construction will work with the designer in house or if it's a consultant project with the Consultant Coordination Section to provide recommendations to the consultant.

This policy should be implemented on all new bridge projects. If you have any questions, please check with me.

Methods Memo No. 79: Integral Abutment Piles
24 July 2003

See C6.5.1.1.1.

Methods Memo No. 9: Battered Pile Capacity and Lateral Load Capacity for Pier Design
9 April 2001

Questions have been raised about whether the capacity of battered piles should be reduced because of the 4:1 batter typically used in pier footings. Based on discussions that I have had with the section leaders, it was decided not to reduce pile capacity for battered piles. The same capacity should be used for the battered piles that are used for the vertical piles.

Another related question that was recently brought up was whether a check of the shear capacity of the piles should be made based on the lateral loads that are applied to the pier. In the past the lateral capacity of the piles has not been a problem for pier design, but the capacity should still be checked. With the longer spans and larger lateral wind and temperature forces that are being applied to the piers, the pile design may be controlled by the lateral loads. If lateral loads are controlling your pile design, inform your section leader. When checking the lateral capacity, use the same allowable lateral capacity for piles that we use in the stub abutment design of 4 kips/pile for wood piles and 6 kips/pile for steel piles.

Steel H-pile examples:

- **(1) Downward load, integral abutment, service load design, for comparison with (2)***
- **(2) Downward load, integral abutment, LRFD, strength limit state, for comparison with (1)**
- **(3) Downdrag, integral abutment, LRFD, strength limit state, for comparison with example in MM No. 87**
- **(4) Uplift, pier pile, service load design, for comparison with (5)***
- **(5) Uplift, pier pile, LRFD, strength limit state, for comparison with (4)**
- **(6) Lateral load, assumed resistance, LRFD, strength and service limit states, for comparison with (7) and (8)**
- **(7) Lateral load, Broms hand computation method, for comparison with (6) and (8)**
- **(8) Lateral load, LPILE software, LRFD, strength and service limit states, for comparison with (6) and (7)**

* Examples (1) and (4) are based on the AASHTO Standard Specifications and the January 2007 Bridge Design Manual. The two examples do not illustrate current LRFD policy.

(1) Downward load, integral abutment, service load design, for comparison with (2)

Given: Bridge meets length, end span length, and skew criteria for integral abutments.
Total abutment vertical service load = 621 kips or 310.5 tons
Use HP 10x57 for integral abutment at 6 ksi.
Prebore 10 feet.
Below prebore: 20 feet firm glacial clay with average N = 11
40 feet very firm glacial clay with average N = 25

Nominal capacity for HP 10x57, friction plus end bearing = AF_a = 16.8*6 = 100.8 kips or 50.4 tons [Steel Construction Manual]

Number of piles = 310.5/50.4 = 6.2, use 7

Design bearing per pile = 310.5/7 = 44.4, say 45 tons

Length per pile [1994 soils information charts]:

Cutoff after driving	1 foot	
Abutment	2 feet	
Prebore	10 feet	
Firm glacial clay	20 feet	(20)(0.7) = 14.0 tons
End bearing in very firm glacial clay (1000 psi = 0.5 tsi)		(16.8)(0.5) = 8.4 tons
Very firm glacial clay	22 feet	(22)(1.0) = 22.0 tons
Total	55 feet, no need to round	44.4 tons

CADD Note E820 on plans: THE DESIGN BEARING FOR THE ABUTMENT PILES IS 45 TONS.

(2) Downward load, integral abutment, LRFD, strength limit state, for comparison with (1)

Given: PPCB D-beam bridge meets length, end span length, and skew criteria for integral abutments with 10-foot prebore.
Total abutment factored vertical load = $\Sigma\eta_i\gamma_iP_i$ = 900 kips (For comparison with Example (1), this is 1.45 x service load.)
Use HP 10x57 for integral abutment at Structural Resistance Level - 1.
Below prebore: 20 feet firm glacial clay with average N = 11
40 feet very firm glacial clay with average N = 25

Nominal structural resistance for HP 10x57, friction plus end bearing at Structural Resistance Level - 1, P_n = 243 kips [BDM Table 6.2.6.1-1]. Nominal resistance also may be limited by Table 6.5.1.1.1-1, but in this example the maximum pile resistance in an integral abutment is 365 kips. The 243 kips controls. (See also the example in the abutment commentary [BDM C6.5.1.1.1].)

Number of piles, n = $\Sigma\eta_i\gamma_iP_i/\varphi_cP_n$ = 900/(0.6*243) = 6.2, use 7

Plan sheet bearing = 50*(6.2/7) = 44.3, say 45 tons [BDM Table 6.2.6.1-1]

Required geotechnical resistance per pile, $P_n = \Sigma\eta_i\gamma_iP_i/\varphi n$ = 900/(0.725*7) = 177 kips

Length per pile [2007 LRFD soils information charts, BDM 6.2.7]:

Cutoff after driving	1 foot	
Abutment	2 feet	
Prebore	10 feet	
Firm glacial clay	20 feet	(20)(2.8) = 56.0 kips
End bearing in very firm glacial clay		(16.8)(2) = 33.6 kips
Very firm glacial clay	22 feet	(22)(4.0) = 88.0 kips
Total	55 feet, no need to round	177.6 kips

CADD Note E820 on plans: THE DESIGN BEARING FOR THE ABUTMENT PILES IS 45 TONS.

(3) Downdrag, integral abutment, LRFD, strength limit state, for comparison with example in MM No. 87

This example makes use of the conditions stated for the examples in Methods Memos Nos. 20 and 87 except that the prebore length was modified from 8 to 10 feet to be in accordance with present policy. (At the time MM No. 87 was written the preferred pile shape for integral abutments in PPCB and CWPG bridges was HP 10x42, but the HP 10x57 shape now is preferred and would be the correct shape for this example.)

Given: Factored abutment dead and live load = 435 kips (This is the 150 ton load used in the methods memos factored with an average load factor of 1.45.)

Soil profile:
- 0-14 feet: fill, medium sand
- 14-18 feet: stiff silty clay
- 18-28 feet: soft-stiff silty clay compressible layer
- 28-38 feet: firm glacial clay
- >38 feet: very firm glacial clay

Grade 50, HP 10x42 piles in 10-foot deep prebored holes

Determine factored downdrag load per pile at strength limit state.

$\gamma DD = 1.0(4*2.4 + 4*1.2 + 10*0.8) = 22.4$ kips

Note that the estimated loads per foot are taken from the “LRFD Driven Pile Foundation Soils Information Chart, English Units” for friction pile [BDM 6.2.7]. For example, the medium sand value of 2.4 kips/foot is the average of fine sand at 2.0 kips/foot and coarse sand at 2.8 kips/foot.

Compute required number of piles, $\eta = 1.0$ implied for all factored loads.

$n(\gamma DD) + \gamma DC + \gamma DW + \gamma(LL+IM) = n(\varphi_c P_n)$

$n(22.4) + 435 = n(0.6*179)$

$n = 5.12$, use 6 piles

P_n is taken from the design manual [BDM Table 6.2.6.1-1].

Compute required nominal geotechnical resistance per pile.

$P_n = \Sigma\eta_i\gamma_i P_i/\varphi n = (6*22.4 + 435)/(0.725*6) = 130.9$ kips

Determine pile length.

Cutoff after driving	1 foot	
Abutment	2 feet	
Prebore	10 feet	
Fill	4 feet	
Stiff silty clay	4 feet	
Soft-stiff silty clay	10 feet	
Firm glacial clay	2 feet	(2)(2.8) = 5.6 kips
Firm glacial clay (below 30 ft)	8 feet	(8)(3.2) = 25.6 kips
End bearing in very firm glacial clay		(12.4)(2)= 24.8 kips
Subtotal		56.0 kips
Required additional resistance = 130.9-56.0 = 74.9 kips		
Required length in very firm glacial clay = 74.9/4.0 = 18.7 feet		
Very firm glacial clay	18.7 feet	(18.7)(4.0) = 74.8 kips
Totals	59.7 feet, round to 60 feet	130.8 kips

Compute theoretical driving resistance values to be included in plan note. These are determined at the service limit state starting with the maximum plan sheet bearing given in BDM Table 6.2.6.1-1.

Dead and live load	= (37)[435/(6*0.6*179)]~~(5.12/6)~~ =	25.0~~31.6~~ tons
Compressible layer and above	= (22.4)/(2*2) =	5.6 tons
Downdrag	= (22.4)/(2*2) =	5.6 tons
Total		36.2~~42.8~~ tons

CADD Note E833 on plans: ABUTMENT PILES ARE DESIGNED TO ACCOMMODATE DOWNDRAG FORCE DUE TO SOIL CONSOLIDATION UNDER THE NEW EARTH FILL. PILES SHALL BE DRIVEN TO 36.2~~42.8~~ TONS BASED ON THEORETICAL DRIVING RESISTANCE. THIS INCLUDES 5.6 TONS OF RESISTANCE IN AND ABOVE THE COMPRESSIBLE LAYER, 5.6 TONS OF RESISTANCE FOR DOWNDRAG FORCES, AND 25.0~~31.6~~ TONS OF RESISTANCE FOR DEAD AND LIVE LOAD BEARING CAPACITY.

Theoretical driving resistance of 36.2~~42.8~~ tons is significantly less than the load that would cause 9 ksi; therefore, a drivability analysis ~~generally~~ would not be required as a check on the design.

This LRFD example required one more pile than the example in Methods Memo No. 87. Actually, considering fractional piles, the increase was about 7% in required pile capacity and with different conditions would not have increased the number of piles. With the increased number of piles, the required pile length was slightly less, and the theoretical driving resistance also was less~~slightly more~~.

(4) Uplift, pier pile, service load design, for comparison with (5)

Given: HP 10x57 embedded 12 inches into footing
Pile designed for end bearing on rock 28 feet below footing
Soft silty clay soil with average N = 3, no scour
10 kips uplift in Service Load Group 6

Allowable capacity for embedment in footing = AF = [(2)(9.99)+(4)(10.2)-(2)(0.565)](12)(0.015) = 10.7 kips [BDM (AASHTO Standard Specifications) 6.6.4.1.3.1]

Allowable capacity for friction bearing, which includes a 75% factor = LF = (0.75)(28)(0.2)(2) = 8.4 kips [1994 soils information charts]

Friction bearing controls, 8.4 kips < 10 kips, NG

(5) Uplift, pier pile, LRFD, strength limit state, for comparison with (4)

Given: HP 10x57 embedded 12 inches into footing
Pile designed for end bearing on rock 28 feet below footing
Soft silty clay soil with average N = 3, no scour
17 kips factored uplift in Strength V limit state (For comparison with the service load design example, this uplift is 1.70 x 10. The 1.70 is an average load factor for piers.)

Factored resistance for embedment in footing = φR_t = (0.45)[(2)(9.99)+(4)(10.2)-(2)(0.565)](12)(0.060) = 19.3 kips [BDM 6.2.6.1]

Factored resistance for friction bearing = $\varphi_{up} R_t$ = (0.6)(28)(0.8) = 13.4 kips [2007 LRFD soils information charts, BDM 6.2.7]

Friction bearing controls, 13.4 kips < 17 kips, NG

(6) Lateral load, assumed resistance, LRFD, service and strength limit states, for comparison with (7) and (8)

Examples (6), (7), and (8) make use of the following given information; however, not all of the information is needed for each example.

Given: Vertical pile, HP 10x57 embedded 12 inches into pier footing (fixed head)
Service limit state loads 6 kips lateral, not sustained; 100 kips vertical
Factored strength limit state loads 10 kips lateral; 145 kips vertical
Lateral load resisted by weak axis bending
Pile designed for end bearing on rock 28 feet below footing
Soft clay soil with average N = 3, single layer, no scour
Soil properties γ = 110 pcf, q_u = 0.75 ksf [Hannigan et al. 2005, C6.2.4.5]

Check vertical load at strength limit state: $\{145\ k\} < \{\varphi_c P_n = 0.6*243 = 145.8\ k\}$, OK
The nominal pile resistance is taken from BDM Table 6.2.6.1-1.

Check lateral load at service limit state: $\{6\ k\} = \{6\ k\}$, OK
The lateral load limit is taken from BDM Table 6.2.6.1-2.

Check lateral load at strength limit state: $\{10\ k\} < \{\varphi P_n = 1.0*18 = 18\ k\}$, OK
The lateral load limit is taken from BDM Table 6.2.6.1-2.

<u>All checks are satisfactory</u>.

(7) Lateral load, Broms hand computation method, for comparison with (6) and (8)

The Broms method is given in the NHI Reference for Course 130082A [Wilson et al. 2005, C6.2.4.5] (Note that the Evans and Duncan method [Brockenbrough 1997, C6.2.4.5] does not consider loads with respect to the weak axis and thus is not applicable for this example.)

Given: See Example (6).

Step 1: Soil type is cohesive (clay).

Step 2: Coefficient of horizontal subgrade reaction. Refer to Table 9-13 [Wilson et al. 2005, C6.2.4.5].

$$K_h = n_1 n_2 80 q_u/b = (0.32)(1.00)(80)(750)/(0.833)(1728) = 13.34 \text{ pci}$$

Step 3: No K_h adjustments are necessary.

Step 4: Pile parameters

$E = 29{,}000{,}000$ psi, $I = 101\ in^4$, $S = 19.7\ in^3$, $f_y = 50{,}000$ psi
$D = 336$ in, $C_s = 1.5$, $M_y = 1{,}477{,}500$ in-lb

Step 5: Embedment factor

$$\beta_h = (K_h b/4EI)^{0.25} = (13.34*9.99/4*29000000*101)^{0.25} = 0.0103$$

Step 6: Dimensionless length factor

$$\beta_h D = (0.0103)(336) = 3.47$$

Step 7: Long or short pile?

$\{\beta_h D = 3.47\} > 2.25$, therefore pile is long.

Step 8: Other soil parameters

$c_u = q_u/2 = 750/2 = 375$ psf

Step 9: Ultimate lateral load for a single pile

$M_y/c_ub^3 = 1477500/(375/144)(9.99^3) = 568$
Figure 9.38 [Wilson et al. 2005, C6.2.4.5] extrapolated: $Q_u/c_ub^3 = 150$
$Q_u = (150)(375/144)(9.99^3) = 39031$ lb
Assuming Q_u is the same as P_n,
{φPn = (1.00)(39.03) = 39.03 kips} > {10 kips}, OK

Step 10: Maximum working load for a single pile

$Q_m = Q_u/2.5 = 39031/2.5 = 15{,}612$ lb, {15.61 k} > {6 k}, OK

Step 11: Working load for a single pile based on deflection

Figure 9.42 [Wilson et al. 2005, C6.2.4.5]: $yK_hbD/Q_a = 3.5$
If y = 0.25 in, $Q_a = (0.25)(13.34)(9.99)(336)/3.5 = 3200$ lb,
Or if $Q_a = 6$ k, {y = (3.5)(6000)/(13.34)(9.99)(336) = 0.47 in} > {0.25 in}, NG

The Broms method predates LRFD and uses a factor of safety of 2.5, but the method obviously permits use of the pile under service or strength limit state lateral loads. It does not, however, give a deflection less than 0.25 inches, which would cause the pile to be rejected.

(8) Lateral load, LPILE software, LRFD, service and strength limit states, for comparison with Example (6) and (7)

Given: See Example (6).

Determine input quantities in lb and inch units
Unit weight = 110/1728 = 0.0637 lb/in^3
Undrained cohesion = (750/2)/144 = 2.60 lb/in^2
Soil strain = ε_{50} = 0.02 [Hannigan et al. 2005, C6.2.4.5]

Input title: HP 10x57 weak axis in soft silty clay, 28 feet to end bearing
Input pile properties: 336, 100, 0, 0
Input pile sectional properties: 1: 0, 9.99, 101, 16.8, 29000000
2: 336, 9.99, 101, 16.8, 29000000
Input loading type: static
Input boundary conditions and loading: 1, shear and slope, 6000, 0, 100000
2, shear and slope, 10000, 0, 145000
Input soil layers: soft clay, 0, 336
Input soft clay: 1: 0.0637, 2.60, 0.02
2: 0.0637, 2.60, 0.02

Output: 0.249 inches lateral deflection at 6-kip service limit state load
564106 in-lb moment at 10-kip factored strength limit state load

Check lateral deflection at service limit state: {0.249 in} < {0.25 in}, OK

Check pile head at strength limit state:
$P_u = 145$ k, $M_{uy} = 564$ in-k, $V_u = 10$ k

$P_r = \varphi_c P_n = (0.7)(0.66^0)(16.8)(50) = 588$ k [AASHTO-LRFD 10.7.3.13.1, 6.9.4.1]
$M_{ry} = \varphi_f M_{ny} = (1.0)(30.3)(50) = 1515$ in-k

$P_u/P_r = 145/588 = 0.247$, therefore
$P_u/P_r + (8/9)(M_{ux}/M_{rx} + M_{uy}/M_{ry}) < 1.0$ [AASHTO-LRFD 6.9.2.2]
$145/588 + (8/9)(564/1515) = 0.591 < 1.0$, OK

$\varphi_v V_n = \varphi_v C V_p = \varphi_v C(0.58 F_{yw} D t_w$ Note that this assumes web will be in shear, which is not true in this example for load causing y-axis bending. Actual resistance will be about twice this value, but AASHTO LRFD Specifications do not cover shear applied to flanges. This shear check is not likely to control even with the lower, incorrect resistance used above.
Check $D/t_w < 1.12(Ek/F_{yw})^{0.5}$ [AASHTO-LRFD 6.10.9.3.1]
$9.99/0.565 < 1.12(29000*5.0/50)^{0.5}$, $17.68 < 60.31$, therefore C = 1.0
$\varphi_v C(0.58 F_{yw} D t_w = (1.0)(1.0)(0.58)(50)(9.99)(0.565) = 163.69$ k > 10 k, OK
See note above regarding actual resistance.

The pile meets the applicable criteria at the service and strength limit states. If the lateral load were increased to the maximum that would meet the combined compression and flexure check, the load would be 22.79 k, more than three times the service load that causes a 0.25-inch deflection.

C6.2.6.2 Concrete-filled steel pipe

In 2009 the Soils Design Section recommended that steel pipe piles generally not be used in soils with consistent N-values greater than 40.

C6.2.6.3 Timber

For timber piles in integral abutment bridges 150 to less than 200 feet (45.700 to less than 61.000 m) in length, pile heads are to be wrapped in carpet padding. The basic detail and note regarding the padding probably were developed in 1965 at the time the bridge on Stange Road just north of 13th Street in Ames was designed, but the detail and note were modified in later years.

The detail (1979) showed the timber pile embedded 2'-0 (600 mm) in the abutment and 3'-0 (1000 mm) rug padding starting at 3 inches (75 mm) from the top of the pile. The top 3 inches (75 mm) of the pile was embedded in the abutment concrete, but below the pile was padded. The pile head was encircled with a spiral with a note: "Spiral at top of each pile 7 turns of #2 bar, 21" diameter, 3" pitch with 2-7/8" C0.69 spacers punched to hold spiral" ("Spiral at top of each pile 7 turns of W5 wire, 535 mm φ, 75 mm pitch with 2-L22 x 22 x 3.2 spacers punched to hold spiral". This metric note is more recent and has a change in spacer.).

The separate note for the padding (1979) is as follows:

> AFTER PILES ARE CUT OFF, THE UPPER 3', EXCEPT AS SHOWN, IS TO BE WRAPPED WITH A DOUBLE THICKNESS OF RUG PADDING HELD IN PLACE BY TACKING WITH GALVANIZED ROOFING NAILS AND WRAPPED WITH #14 GAGE GALVANIZED WIRE AT 4" PITCH. CARE IS TO BE TAKEN NOT TO DAMAGE PADDING WHEN PLACING CONCRETE. RUG PADDING MAY BE EITHER OF THE FOLLOWING:
>
> (1) HAIR AND JUTE RUG PADDING, RUBBERIZED ON BOTH SIDES, AND WEIGHING NOT LESS THAN 47 OZ. PER SQ. YD.
> (2) BONDED URETHANE OR BONDED POLYFOAM WITH A MINIMUM DENSITY OF 5 LBS. PER CU. FT. AND SHALL BE AT LEAST ½ IN. THICK. (MATERIAL LESS THAN ½ IN. IN THICKNESS MAY BE USED, BUT WILL REQUIRE ADDITIONAL WRAPS FOR A TOTAL OF AT LEAST ONE INCH.)

Methods Memo No. 9: Battered Pile Capacity and Lateral Load Capacity for Pier Design
9 April 2001

See C6.2.6.1.

Service load design for timber piles in piers and stub abutments
The Iowa DOT Standard Specifications [IDOT SS 4165.03, H] give dimension specifications for timber piles ranging from less than 20 feet to over 60 feet. However, office practice has limited timber piles to lengths of 20 to 55 feet.

Past office practice has implied that timber piles only need to be checked for geotechnical capacity by limiting the capacity for pier and stub abutment piles to 20 tons for piles 20 to 30 feet long and to 25 tons for piles 35 to 55 feet long. These capacities run counter to the axial structural capacity, which is based on the tip area of the pile and which decreases with pile length as the following computations show.

- 20- to 35-foot pile, minimum tip diameter = 8 inches [IDOT SS 4165.03, H], resulting tip area = 50.27 in^2, allowable working stress for douglas fir or southern pine = 1200 psi [AASHTO Table 4.5.7.3A], load duration factor for vehicle live load = 1.15 [AASHTO Table 13.5.5A], and capacity = 69.37 kips or 34.69 tons
- 40- to 55-foot pile, minimum tip diameter = 7 inches [IDOT SS 4165.03, H], resulting tip area = 38.48 in^2, allowable working stress for douglas fir or southern pine = 1200 psi [AASHTO Table 4.5.7.3A], load duration factor for vehicle live load = 1.15 [AASHTO Table 13.5.5A], and capacity = 53.10 kips or 26.55 tons

Within the design rules and specifications, the structural capacity for pier and stub abutment piles should not control. (The piles also should be checked for dead load alone with a load duration factor of 0.90. Because of the relative magnitudes of live and dead load this case typically does not control.)

It is interesting to note that *Foundation Soils Information Chart, Pile Foundation* ("Blue Book") assumes a relatively large end area of 72 in^2, which correlates with a tip diameter of 9.57 inches, more than required by the standard specifications [IDOT SS 4165.03, H]. A note requires that geotechnical bearing be adjusted for a different tip dimension, which generally would require reduction of the tabulated values. In Appendix B, Table 5 of the "Blue Book" the average wood pile diameter is given as 10 inches, and after Table 9 the timber pile design bearing is given as 900 psi. Even though the average timber pile diameter is about 10 inches it would appear that the "Blue Book" is not consistent with the Iowa DOT standard specifications.

Based on the example for downdrag in Appendix A of the "Blue Book" it appears that downdrag could easily cause overstress of a timber pile during driving. To avoid overstress the office in the past had a 40-ton driving limit. Generally it would seem inadvisable to use timber piles when they will be subjected to downdrag.

Service load design for piles for integral abutments in prebored holes
Past office practice limited timber piles for integral abutments to 20 tons [BDM 6.2.6.3]. The reason stated for this limit was indeterminate bending stresses (which are likely to occur for bridges less than 150 feet long for which the pile heads are not wrapped in carpet padding).

Another possible reason for the limit is reduction in capacity due to lack of lateral support in a prebored hole. Prebored holes are required for bridges longer than 130 feet. At the time the 20-ton rule was instituted the typical prebore was 8 feet. If one makes a few assumptions, the column stability factor reduction can be estimated for a pile in an 8-foot prebored hole.

- Assume pile is fixed 4 feet below bottom of prebore. This distance will vary depending on stiffness of soil but generally is in the range of 2.5 to 4 feet. The equations for preliminary design in the AASHTO LRFD Specifications indicate that the distance would exceed 4 feet only for very loose submerged sand [AASHTO-LRFD 10.7.3.13.4].
- Assume K-factor for slenderness is 0.8. This assumption is more liberal than a flagpole assumption (K=2.1), but reasonable considering the overall superstructure, fixity at one pier, and embankment restraints due to pavement and wing walls. This assumption also is consistent with the ISU research report on which design for integral abutment piles is based [Greimann et al 1987].

- Assume average pile diameter for the top 12 feet of pile is 8.7 inches (interpolated from Southern Pine Foundation Piling—Specified Tip Circumferences with Corresponding Minimum Butt Circumferences, ASTM D25). The equivalent square area of the pile then would have a side dimension of 7.71 inches.
- Assume Service Load Group I, and therefore load duration factor = 1.15 for vehicle live load [AASHTO Table 13.5.5A].
- Assume F_c = 1200 psi for Douglas fir or southern pine under wet or dry conditions [AASHTO 4.5.7.3].
- Assume K_{cE} = 0.418 as for glulam timber [AASHTO 13.7.3.3]. The AASHTO Standard Specifications do not specifically cover timber piles, but the AASHTO LRFD Specifications list the same value of K_{cE} for glulam and round piles [AASHTO-LRFD 8.8.2].

With the assumptions above, the column stability reduction is about 0.89, which would reduce the pile minimum structural capacity to 23.63 tons but not less than 20 tons. The additional capacity provides some margin for bending stresses that occur as the bridge expands or contracts. Additionally the AASHTO Service Load Groups that include temperature loads have a stress increase of 125% or 140%.

A separate check with the assumptions above but with a 10-foot prebored hole, a 200-foot long bridge, and a moment due to thermal expansion under Load Group IV gave a performance ratio of 0.542, indicating that a timber pile would be acceptable considering column stability and bending.

Geotechnical design for timber piles under LRFD

The traditional limits for geotechnical capacity of timber foundation piles have served well, and there appears to be no reason to alter the limits. Therefore, to fit the service load design limits to LRFD, the office has adjusted service limits by an average load factor, γ, of 1.45 divided by a resistance factor, φ, of 0.725. The adjustment computes to 2.00. For stub abutments and piers the maximum LRFD nominal resistance will be 80 kips for piles 20 to 30 feet long and 100 kips for piles 35 to 55 feet long. Also, the maximum LRFD nominal resistance for timber piles in integral abutments will be 80 kips. As a general check on these limits, the "Blue Book" states that the majority of the timber pile load tests experienced yield at no more than 75 tons (150 kips) and that the ultimate load (used in design) should not exceed 60 tons (120 kips).

Determining pile length can follow the usual office procedures with φ = 0.725.

LRFD structural design

For a fully embedded pile the usual design procedure is to check axial compression at the pile tip. The various factors necessary for the check are given in the AASHTO LRFD Specifications [AASHTO-LRFD 8.4 and 8.5.2.2].

In cases where a timber pile is unsupported over some length, the pile should be checked structurally using the information in Chapter 8 of the AASHTO LRFD Specifications.

If the 80-kip limit for maximum nominal geotechnical resistance for an integral abutment pile is followed, the designer need not check an integral abutment timber pile in a prebored hole 10 feet deep or less.

Driving limits

Damage due to hard driving has been a concern for timber piles. When the office often used timber piles in the early 1970s (and earlier) timber piles were not to be used in glacial clays with N greater than or equal to 30. In 1982 the guideline was revised downward so that timber piles were not to be used in any soils in which N exceeded 25. In 1991 a guideline required that timber piles not be driven for more than 40 tons. That guideline exceeded the maximum allowable design load and thus may have been a limit used in the field to prevent overdriving.

References

American Forest and paper Association (AF&PA) (2001). *National Design Specification, ANSI/AF&PA NDS-2001.* AF&PA, American Wood Council. Washington, DC.

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Greimann, L.F., R.E. Abendroth, D.E. Johnson., and P.B. Ebner. (1987). *Pile Design and Tests for Integral Abutments, Final Report.* Department of Civil Engineering, Iowa State University. Ames, IA.

Treated timber pile examples:

(1) Downward load, pier, service load design, for comparison with (2)
(2) Downward load, pier, LRFD, for comparison with (1)

(1) Downward load, pier, service load design, for comparison with (2)

Based on the designer's experience it appeared that relatively short timber piles would be required for which the maximum load would be 20 tons. Therefore, the designer proportioned the footing and pile arrangement to limit the pile load to 20 tons or less.

Given: Vertical service load for Load Group I = 38 kips or 19 tons
Below pier footing: 10 feet firm glacial clay with average N = 11
40 feet very firm glacial clay with average N = 24

Note: Timber piles should not be driven in soils with an N > 25. The very firm glacial clay is close to this limit.

Note: The load duration factor, C_D, will vary by load group. This example assumes that Load Group I is the critical case, but another load group could control the design.

Length per pile [1994 soils information charts]:

Cutoff after driving	1 foot			
Pier footing	1 foot			
Firm glacial clay	10 feet	(10)(0.6)	=	6.0 tons
End bearing in very firm glacial clay			=	7.6 tons
Very firm glacial clay	8 feet	(8)(0.7)	=	5.6 tons
Total	<u>20 feet</u> (no need to round)			19.2 tons

Structural check at pile tip

Fc = 1200 psi for douglas fir or southern pine [AASHTO Table 4.5.7.3A]
C_D for vehicle live load = 1.15 [AASHTO Table 13.5.5A]
Minimum tip diameter for 20-foot pile = 8 inches [IDOT SS 4165.03, H], tip area = 50.27 in^2
$P_C = F'_C A = (1200)(1.15)(50.27) = 69{,}373$ lb = 69.37 kips = 34.69 tons
$P_C > 19$ tons, OK

CADD Note E720 on plans: THE DESIGN BEARING FOR THE PIER PILES IS 19 TONS.

(2) Downward load, pier, LRFD, for comparison with (1)

Given: Total pier factored vertical load for Strength I = $\Sigma\eta_i\gamma_i P_i$ = 55 kips (For comparison with Example (1), this is 1.45 x service load, rounded.)
Below pier footing: 10 feet firm glacial clay with average N = 11
40 feet very firm glacial clay with average N = 24

Note: Timber piles should not be driven in soils with an N > 25. The very firm glacial clay is close to this limit.

Note: The time effect factor, C_λ, will vary by limit state. This example assumes that Strength I is the critical case, but another limit state could control the design.

Required geotechnical resistance for the pile, $P_n = \Sigma\eta_i\gamma_i P_i/\varphi = 55/0.725 = 76$ kips

Length for the pile [2007 LRFD soils information charts, BDM 6.2.7]:

Cutoff after driving	1 foot	
Pier footing	1 foot	
Firm glacial clay	10 feet	(10)(2.4) = 24.0 kips
End bearing in very firm glacial clay		= 30.4 kips
Very firm glacial clay	8 feet	(8)(2.8) = 22.4 kips
Total	<u>20 feet</u> (no need to round)	76.8 kips

Structural check at pile tip

Fco = 1.20 ksi for southern pine < douglas fir [AASHTO-LRFD Table 8.4.1.3-1]
$\varphi = 0.90$ for compression parallel with grain [AASHTO-LRFD 8.5.2.2]
$C_{KF} = 2.5/\varphi = 2.5/0.90 = 2.78$ [AASHTO-LRFD 8.4.4.2]
C_λ for Strength I = 0.8 [AASHTO-LRFD Table 8.4.4.9-1]

Minimum tip diameter for 20-foot pile = 8 inches [IDOT SS 4165.03, H], tip area = 50.27 in^2
$\varphi P_n = \varphi F_C A = (0.90)(1.20)(2.78)(0.8)(50.27) = 120.74$ kips
$\varphi P_n > 55$ kips, OK

Plan sheet bearing = 55/1.45 = 37.9 kips = 18.97 tons, say <u>19 tons</u>

CADD Note E720 on plans: THE DESIGN BEARING FOR THE PIER PILES IS 19 TONS.

C6.2.6.4 Prestressed concrete

The office has relaxed limitations for use of prestressed concrete piles since the 1970s. In 1975 a memo recommended that concrete piles not be used in very firm glacial clay or very firm sandy glacial clay when N values exceed 20. The limit was set due to driving experience. Since the late 1980s the rule has been revised to not drive prestressed piles more than 10 feet through soils with N values greater than 40. In 1995 a memo advised that prestressed piles should tip out in soils with N values from 25 to 40 and which do not contain boulders. In 2009 the Soils Design Section recommended that prestressed concrete piles generally not be used in soils with consistent N-values greater than 30 to 35.

In 1995 the office set a service load design bearing capacity of 50 tons (which a few years earlier had been 40 tons) for a 12-inch square prestressed concrete pile. It was unclear as to whether the limit was structural or geotechnical; it has been assumed to be structural. The limit has been fitted to a LRFD nominal structural resistance using an average load factor of 1.45 and a resistance factor of 0.75 for a compression controlled section [AASHTO-LRFD 5.5.4.2.1] as follows:

(50)(2)(1.45)/0.75 = 193 kips, round to 200 kips.

C6.2.7 Geotechnical resistance charts

Appendix for obsolete and superseded memos